Paper Title:

Vertical Drains Under a High Embankment on a Very Soft Till

Oxford Brook, Miramichi, New Brunswick

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Abstract

The New Brunswick Department of Transportation proposed to construct a 13 metre high approach embankment leading to a single span overpass structure founded on spread footings.

Preliminary investigation of the subsurface conditions revealed that the site is underlain by up to 22 metres of compressible soils. Much of this soil was glacial till, which would normally suggest no settlement or stability problems. However, our detailed review of the existing soils information has shown that this glacial till is a so-called flow till or water-laid till, which is typically a relatively soft soil. Significant settlements of up to 600 mm were anticipated. To further compound this problem, the compressible layer was significantly less thick at the north abutment, and the differential settlement between the north and south abutments could be as much as 0.3 metres.

The calculated time required to reach 90% consolidation was in the order of 20 years. This magnitude of settlement over a 20-year period was considered unacceptable by the NBDOT.

To remedy this problem, vertical drains were used to expedite the rate of settlement. The drains were installed in a triangular pattern with spacings of 1.5, 3.0 and 5.0 metres. The spacing was smallest in the area of the proposed bridge abutments and was increased away from the abutments. Four settlement plates and three piezometers were installed to monitor the settlement and pore water pressure during and after construction of the embankment.

Settlement measurements showed primary consolidation ended in less than a year, agreeing well with predicted values.

1 Introduction

In the spring of 2003 NBDOT began the process of constructing an interchange at Route 8 and Route 425; which forms part of the Route 8 Miramichi Bypass. The bypass will provide a controlled access highway designed to a RAU 110 standard. The bypass will no longer force motorists travelling on Route 8 to drive through downtown Miramichi on Route 8, also known as the King George Highway. This bypass will alleviate traffic flow within the city. Route 8 is presently the main route connecting Fredericton, New Brunswick, to the Miramichi, located approximately 180 kms southwest. This route intersects Route 425 near the Oxford Brook area, located to the southwest of the Miramichi.

The new proposed bypass alignment is illustrated in Figure 1.1.

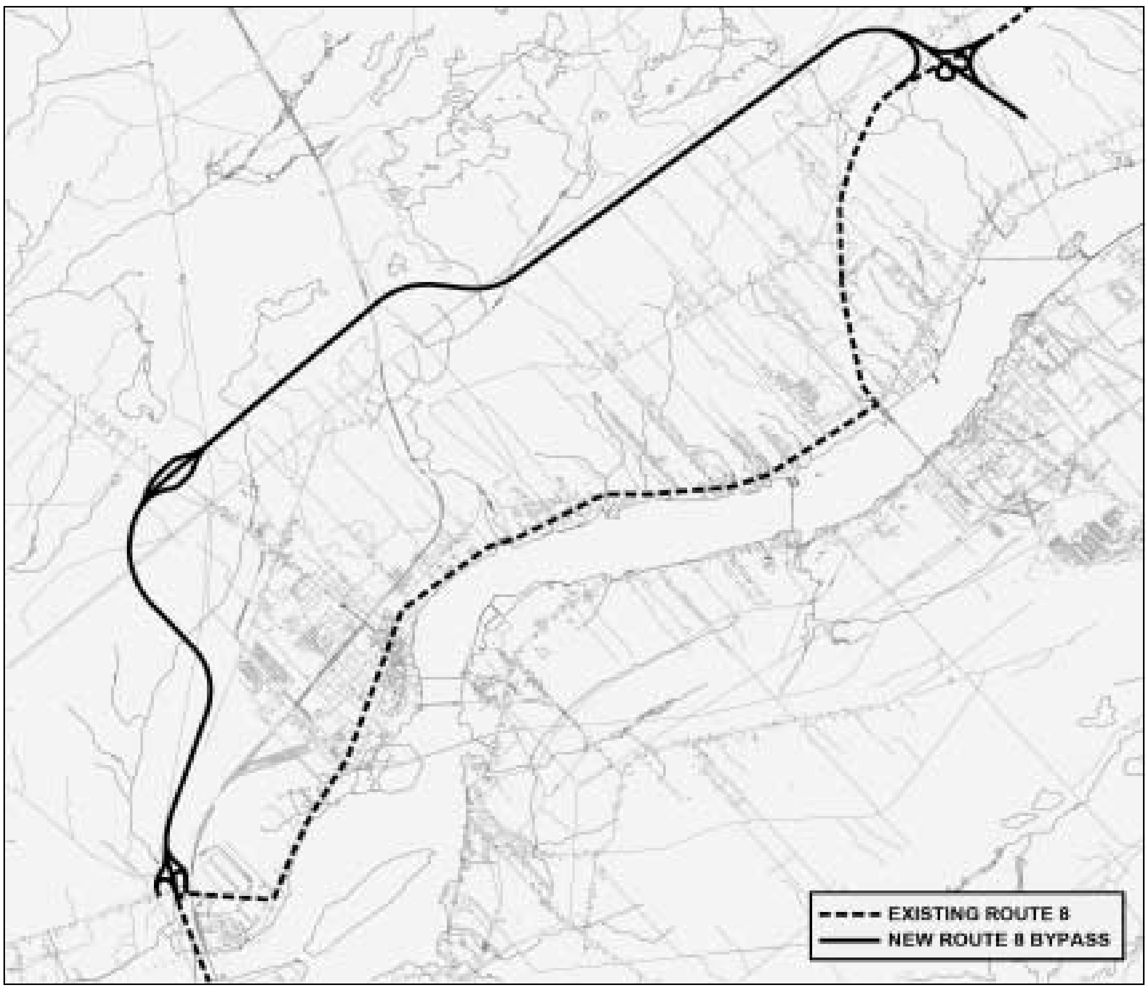

Figure 1.1: *Existing route and new proposed Miramichi bypass*

The bypass project requires that approximately 13 metre high approach embankments be constructed along Route 8, leading up to a single span bridge over the existing Route 425. The single span bridge abutments will be supported by spread footings founded on approximately 5 metres of engineered fill. NBDoT proposed to construct the embankment in the spring of 2004 and to start bridge construction in the spring of 2005. The bypass project is scheduled to be complete by the fall of 2006.

Following a geotechnical investigation, it was found that a thick deposit of compressible soil, as much as 22 m, covered a majority of the Oxford Brook bypass area. Much of this soil was glacial till, which would normally suggest no settlement or stability problems. However, our detailed review of the existing soils information showed that this glacial till is a so-called flow till or water-laid till, in this case a relatively soft soil. Significant settlements up to 600 mm are anticipated under the centreline of the 13 (±) metre high embankment. To further compound this problem, the compressible layer is approximately 5 metres thinner at the north abutment than at the south abutment. A differential settlement of 0.2 to 0.3 metres is expected between the north and south abutments.

The New Brunswick Department of Transportation required that the compressible soils underlying the proposed overpass structure and approach embankments reach an acceptable degree of consolidation within a one-year period. Based on laboratory consolidation tests, the time required for the soft soils to reach 90% consolidation, provided that the soft compressible soils were simply loaded with embankment fill, was predicted to be in the order of 20 years. This was an unacceptable length of time as construction of the overpass was scheduled to begin in May of 2005. To remedy this problem prefabricated vertical drains (PVD) were proposed to expedite the consolidation process. It should be noted that PVD's are not typically installed in a till soil, which contain gravel and cobble-sized particles. Therefore, installation of these drains could be difficult.

2 Soil Conditions

NBDoT undertook a geotechnical field investigation to characterize the subsurface soil conditions across the site. Between 1999 and 2004, 21 conventional boreholes were put down along the proposed approach embankment and at the proposed overpass abutments. Several Shelby tube samples were recovered for laboratory consolidation and triaxial testing.

The borehole information shows that the site is covered by a by a 2 to 5 metre thick layer of sand and gravel, underlain by 0 to 6 metres of clayey silt, which in turn is underlain by up to 22 metres of glacial till. Depth to bedrock ranges from zero (bedrock outcropping west of the north abutment) to as much as 24 metres (south abutment).

The rock profile from north to south is shown on Figure 2.1. Typical cross sections of the soils conditions under the north and south abutments are shown in Figures 2.2.

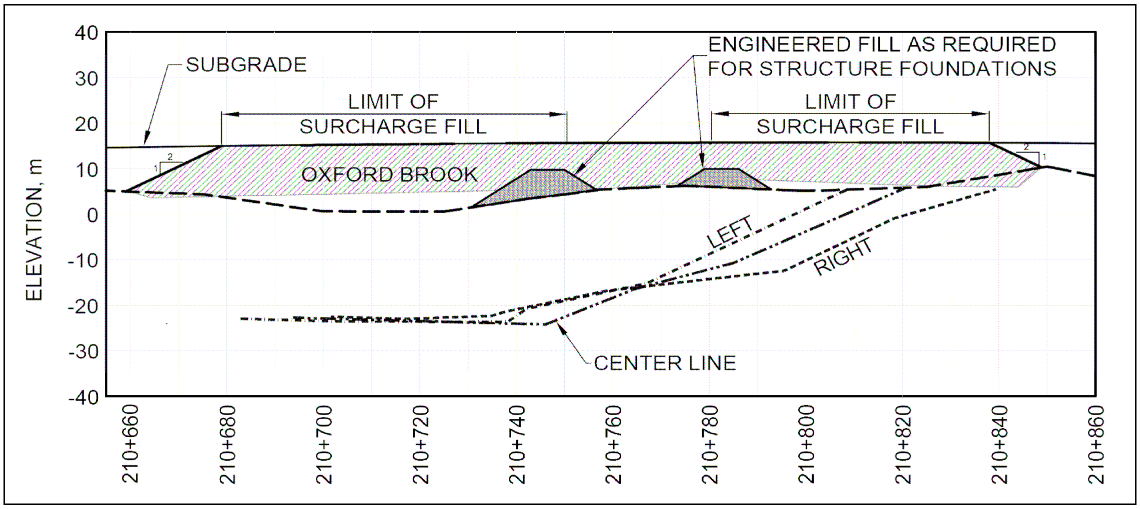

Figure 2.1: *Bedrock profile*

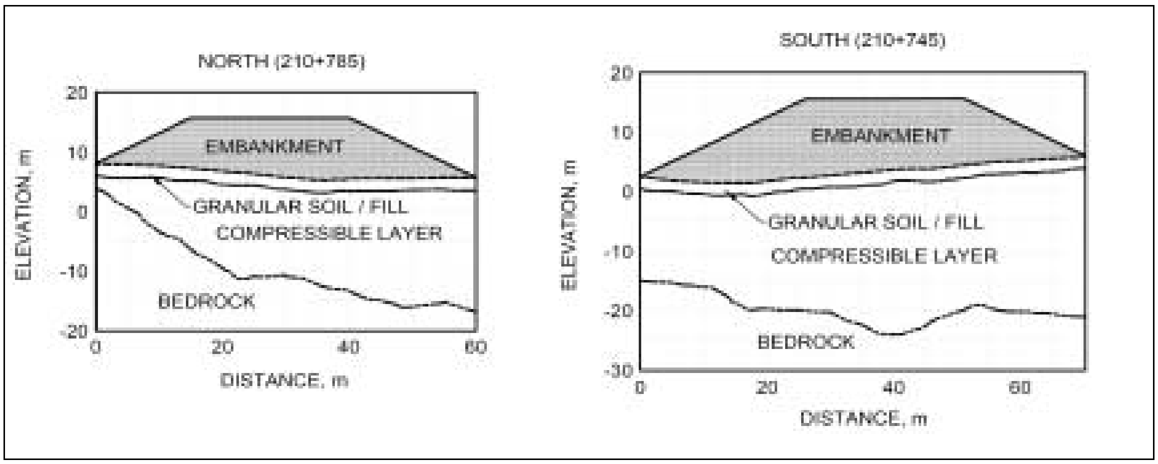

Figure 2.2: *Abutment cross-sections*

Standard Penetration Test (SPT) N-values recorded in the till typically ranged between 8 and 12, but values as low as 2 to 4 were recorded. The moisture content ranged from 13 to 30%, averaging 18%. The plastic and liquid limits of the till ranged from “non plastic” to 20% and “non plastic” to 30%, averaging 14% and 22%, respectively.

The till is composed of an average of 11% gravel, 36% sand, 22% silt, and 31% clay sized particles. It should be noted that cobbles were encountered throughout the till layer, as is typical in most till deposits.

NBDoT conducted laboratory consolidation tests on the till. The compression index (C_c) ranged from 0.07 to 0.11, and the coefficient of consolidation (c_v) ranged from 0.1 to 1.2 mm^2/s. The initial void ratio (e_o) ranged from 0.41 and 0.67.

3 Settlements

Expected settlements of the embankment at the north and south bridge abutments were estimated from laboratory consolidation data. An average compression index and initial void ratio of C_c = 0.09 and e_o = 0.67, respectively, were used for settlement calculations.

The settlement at the centerline of the north abutment where the compressible till layer is approximately 16 metres thick was estimated to be in the order of 400 mm under an 11 metre high embankment. At the centerline of the south abutment where the compressible till layer is approximately 22 metres thick, the settlement was estimated to be in the order of 600 mm under a 13 metre high embankment.

For rate of settlement calculations, the value of the coefficient of consolidation c_v for the flow till was taken to be in the range of 0.25 to 0.35 mm^2/sec or about 10 m^2/yr as determined on the basis of the time required to reach 90% consolidation in the laboratory. This would give a time of about 20 years to reach 90% consolidation in the field. The corresponding time to reach 50% settlement would be about 5 years. These estimates are subject to certain limitations, considering for example the minute sample size (20 mm thick and 50 mm diameter) as compared to the 16 to 22 metre thick by about 80 m wide by 100 metre length layer in the field.

The above settlements and estimated time to reach 90% consolidation were considered unacceptable by the NBDOT, thus measures to expedite the settlement were required.

4 Prefabricated Vertical Drains

The NBDOT required that the compressible soils underlying the abutment locations reach 90% consolidation within a one-year period. To achieve this goal we opted to use prefabricated vertical drains (PVD). By installing vertical drains, settlement is expedited through a drastic decrease in the length of the drainage path, allowing excess pore water pressure to dissipate more quickly (Figure 4.1).

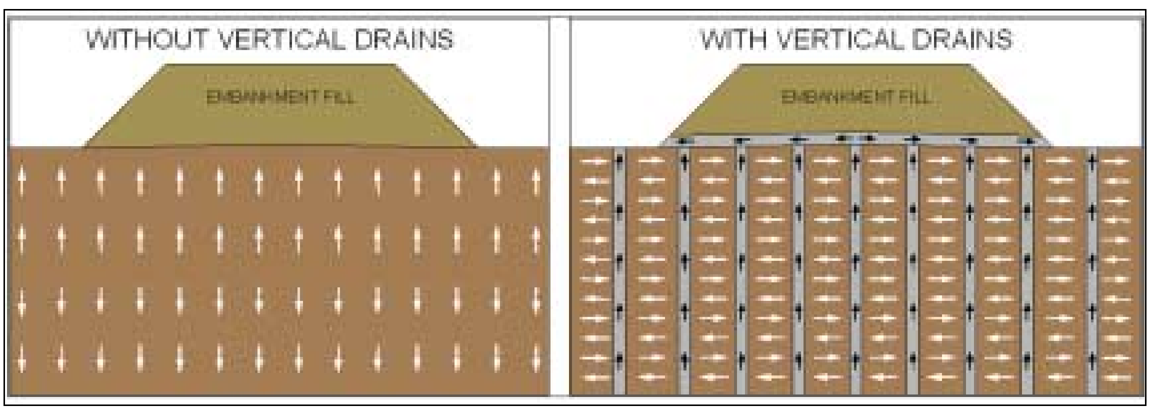

Figure 4.1: *Drainage path length*

The spacing and configuration of the drains were optimized using Hansbo's Method [1]:

$$t = \frac{D^2}{8c_h}\left[\frac{\ln\left(D/d\right)}{1-\left(d/D\right)^2} - \frac{3-\left(d/D\right)^2}{4}\right]\ln\left(\frac{1}{1-U_r}\right)$$

where

t = time to reach 90% consolidation
U = degree of consolidation (90% in this application)
c_v = coefficient of horizontal consolidation ($c_v \approx c_h$)
$D = 1.05S$ *for triangular spacing*

$$d = \frac{2\times drain\ width + 2\times drain\ thickness}{\pi}$$

In order to use Hansbo's method, the properties of the vertical drains and the value of the radial coefficient of consolidation for the compressible layer are required. As tests to determine the coefficient of radial consolidation were not done, an assumed value of $c_h \approx c_v$ was used. Calculations were done assuming triangular spacing and an average c_v value of 0.30 mm^2/s. Drain properties are given below.

Drain	Typar Filter	Width	Thickness	Assumed % Voids
Mebra Drain MD88	3401	100 mm	3.4 mm	92

Table 4.1: *Vertical Drain Properties*

Spacings of 1.5, 3.0, and 5.0 metres in a triangular pattern were selected. The spacing was smallest at the abutments (1.5 metres) and as the drains progressed farther from the proposed abutment locations, the spacing was increased to 3.0 and 5.0 metres. The idea was to reduce costs and provide a uniform transition from the approach embankment to the overpass abutment.

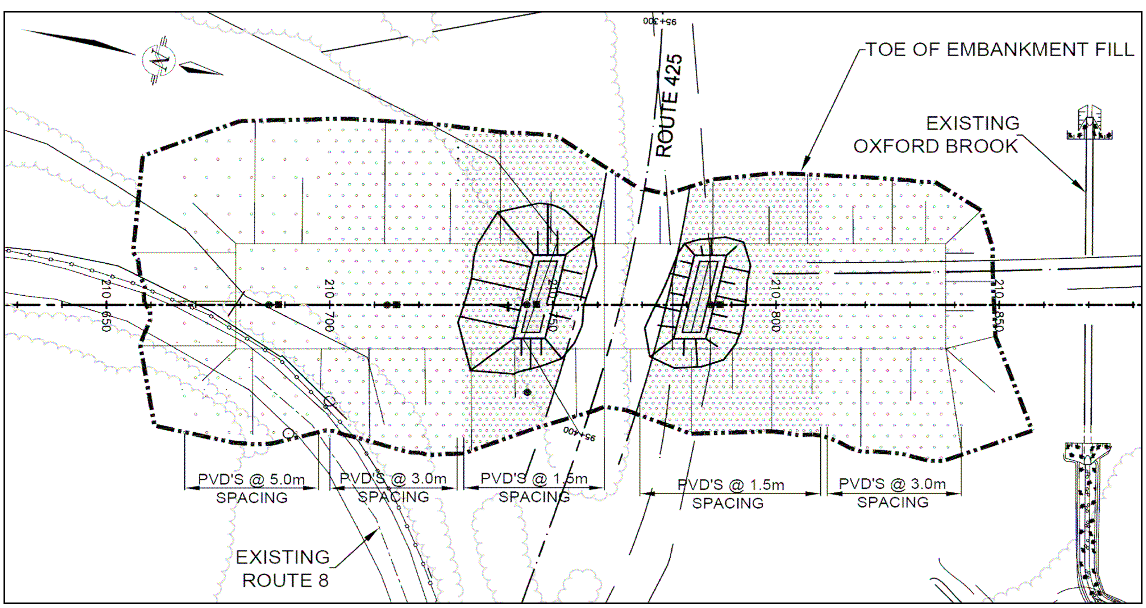

Figure 4.2: *Plan view of PVD installation pattern*

The computed times to achieve 90% consolidation using PVD's at various spacings and locations across the site is presented in Table 4.2.

Location	PVD Spacing (m)	t_{90} (years) (c_h = 0.3mm^2/s)
Sta 210+785 (North Abutment)	1.5	0.18
Sta 210+745 (South Abutment)	1.5	0.18
Sta 210+715	3.0	0.94
Sta 210+690	5.0	3.04

Table 4.2: *t_{90} from Hansbo's Method*

It should be noted that the average value of c_v = 0.3 mm^2/s was used, while actual measured values ranged from 0.1 to 1.2 mm^2/s.

The Mebra Drain MD88 prefabricated vertical drains were installed by means of a steel mandrel housed within a large vertical mast. This was attached to a 550LC sized Hitachi excavator. The excavator was set up to push the mandrel, which housed the vertical drain, through the till deposit. The mandrel would then be pulled back, and a special anchor plate would remain in the ground holding the drain in place.

Figure 4.3: *Excavator attached to mast*

The site was initially graded to have a 0.5 to 4% slope across the embankment bottom. Prior to installation of the drains, a 300 mm thick sand drainage layer was spread cross the site. This layer acted as a drainage boundary, allowing pore water travelling up the drains to dissipate horizontally. Augering equipment attached to an excavator was required to pre-auger through the sand and gravel layer which covers the soft soil. This was done at each vertical drain location, as the mandrel could not be pushed through the granular layer.

Figure 4.4: *Pre-augering drain locations*

During the installation of the prefabricated vertical drains, problems were encountered while attempting to push the mandrel through the till layer, as this soil is somewhat stiffer than typical soft soils where PVD are installed. In addition, the high percentage of sand and gravel particles further compounded the problem. As a result of this we were unable to push the steel mandrel more than a few metres into to till layer until a water hose was installed on the inside of the steel mandrel, allowing a mist of water to exit at the tip of the mandrel. This technique allowed the drains to be pushed through the 16 to 22 metre thick till layer with relative ease.

A photo of the water mist exiting the tip of the mandrel is shown in Figure 4.5 below.

Figure 4.5: *Mandrel lubrication*

Figure 4.6: *Vertical drain installation*

5 Instrumentation

In order to monitor the performance of the prefabricated vertical drains a total of four settlement plates and three piezometers were installed. The settlement plates were standard settlement plates, constructed out of a two foot by two-foot square ¾ inch thick plywood base and a two-inch diameter steel pipe connected to the base at the centre. Settlement plates were installed at the north and south abutments (sta 210+785 and 210+745 respectively), and under the southern approach embankment at sta 210+715, and 210+690.

The piezometers used were model PWP vibrating wire piezometers. They were installed at three of the settlement plate locations (sta 210+785, 210+745, and 210+715) at depths of 3.86, 7.26, and 5.00 metres respectively.

6 Field Data

Measurements of settlement and pore pressure were taken during and after construction of the embankment.

The excess pore pressure developed within the compressible soil during embankment construction dissipated to essentially zero approximately 50 days after the embankment fill was complete. Following this, secondary creep settlements were observed. The rates of plastic secondary creep (c_α) at the north and south abutments were 0.04 and 0.05%, respectively, determined as the slope of the settlement curve after dissipation of excess pore pressures.

Total settlements of 320 and 620 mm were observed approximately 10 months after embankment construction at the north and south abutments, respectively.

Settlements and pore pressures under the north and south abutments are plotted in Figures 6.1 and 6.2.

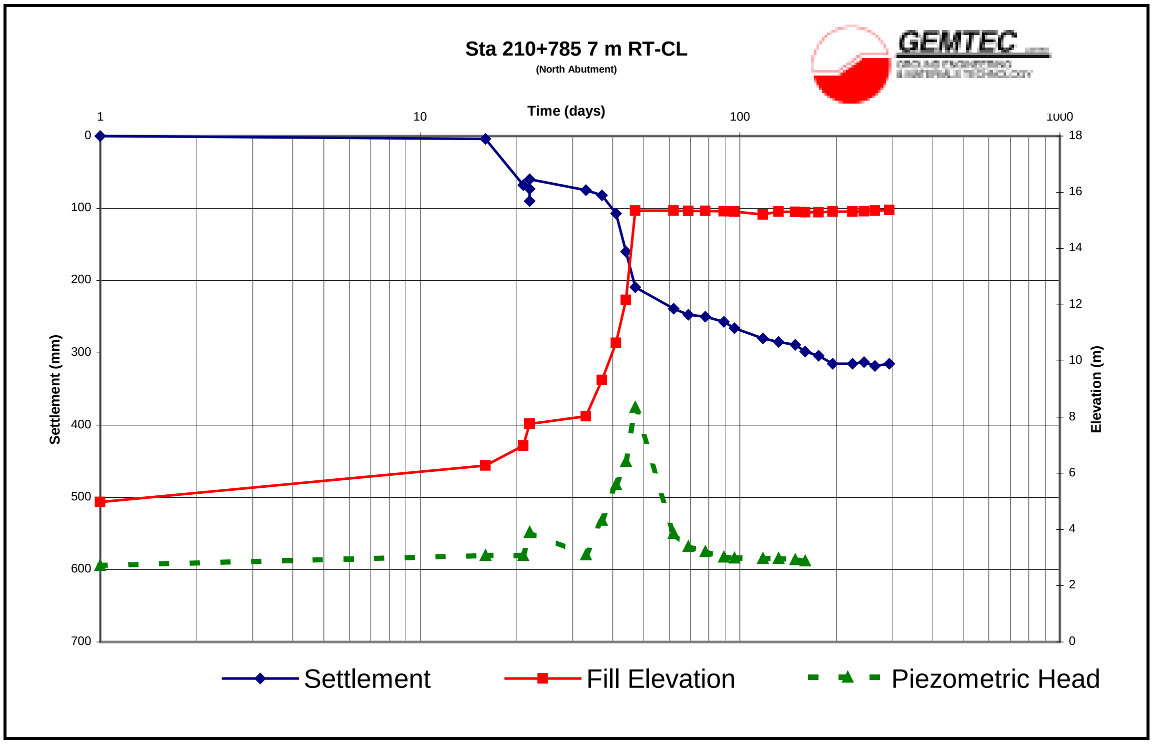

Figure 6.1: *Settlement and pore pressure data at the north abutment (sta 210+785)*

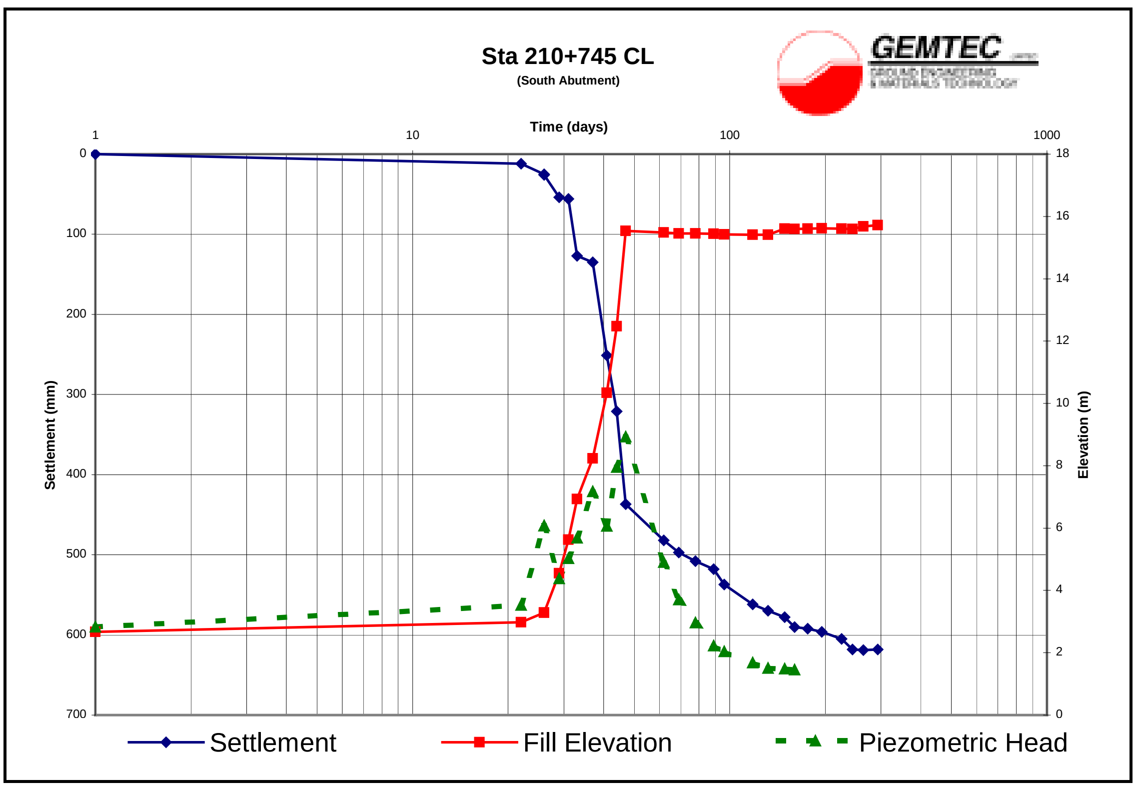

Figure 6.2: *Settlement and pore pressure data at the south abutment (sta 210+745)*

7 Discussion

As shown in Figures 6.1 and 6.2, the excess pore water pressure dissipated rapidly, indicating that the PVD's performed well.

The computed times to reach t_{90} using PVD's (Hansbo's Method) compared well with field measurements. The times required to reach t_{90} at the four monitored stations fell within the predicted limits (based on the range of c_v values determined through laboratory testing, 0.1 to 1.2 mm^2/s) as shown in Figure 7.1.

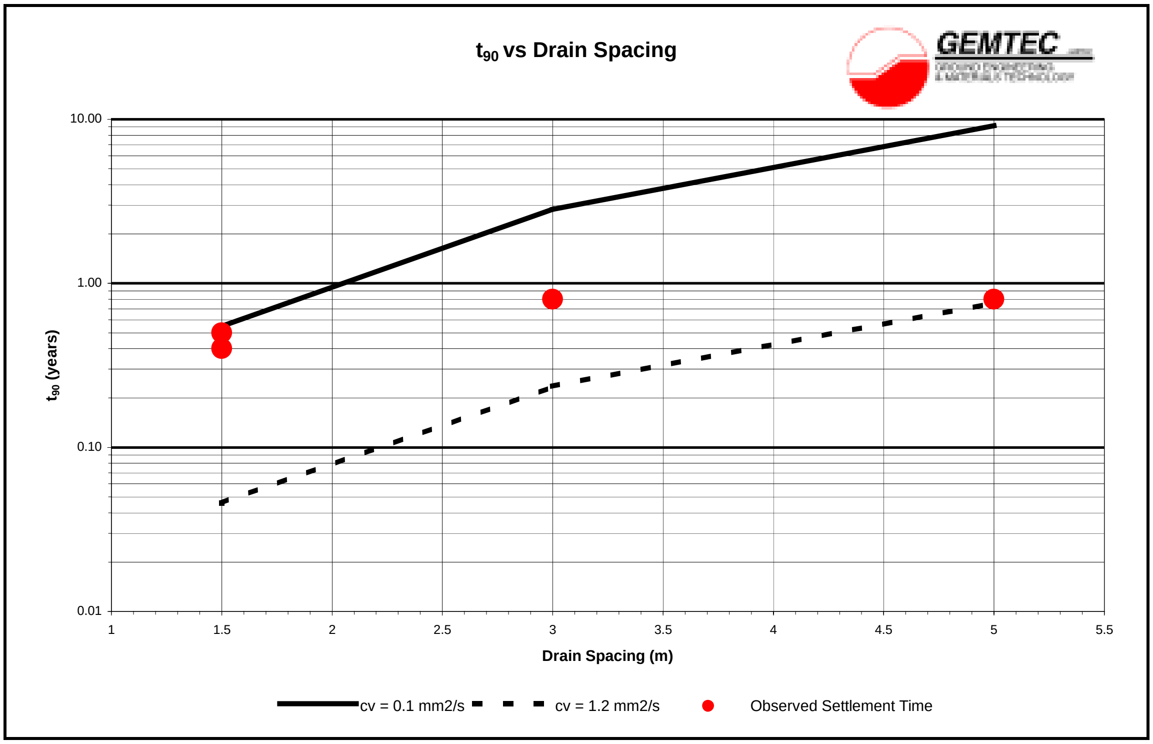

Figure 7.1: *t_{90} - Predicted vs actual*

Computed settlements were also found to be in good agreement with those measured in the field, as shown in Table 7.1.

Location	Predicted Settlement (mm)	Measured Settlement June 2004 to March 2005 (mm)
North Abutment (sta 210+785)	400	320
South Abutment (sta 210+745)	600	620

Table 7.1: *Summary of total computed and measured settlement*

Primary consolidation being completed, the proposed Route 8 - Route 425 overpass could be constructed on conventional spread footings. As of April 2006 there has been minimal settlement (i.e. less than 2 mm) of the bridge deck.

8 Conclusion

Prefabricated vertical drains were used successfully to expedite settlement in the thick deposit of soft till soils underlying the Oxford Brook area of the Route 8, Miramichi bypass. The total time required for the soft till soils to reach 90% consolidation was reduced from a predicted 20 years to approximately 50 days. The magnitude of settlement also agreed well with computed values.

The use of prefabricated vertical drains eliminated the need for piles and therefore also the long-term maintenance that would have been required because of differential settlements between the approach embankments and the piled abutments.

Figure 8.1: *Completed overpass structure looking east*

References

[1] Koerner, R.M., 1998. “Designing with Geosynthetics”, 4th Edition, Prentice-Hall